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2012 Emilia earthquake, Italy: Reinforced Concrete buildings response

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Abstract

Data of the Italian National Institute of Statistics are collected aimed at characterizing Reinforced Concrete (RC) building stock of the area struck by the 2012 Emilia earthquake (number of storeys, age of construction, structural typology). Damage observations, collected right after the event in reconnaissance reports, are shown and analyzed emphasizing typical weaknesses of RC buildings in the area.

The evolution of seismic classification for Emilia region and RC buildings' main characteristics represent the input data for the assessment of non-structural damage of infilled RC buildings, through a simplified approach (FAST method), based on EMS-98 damage scale. Peak Ground Acceleration (PGA) capacities for the first three damage states of EMS-98 are compared with registered PGA in the epicentral area. Observed damage and damage states evaluated for the PGA of the event, in the epicentral area, are finally compared. The comparison led to a fair agreement between observed and numerical data.

Keywords: Emilia earthquake, infilled RC buildings, damage states, non-structural damage, FAST method

1. INTRODUCTION

On the 20th of May 2012 a magnitude (M_w) 6.0 earthquake struck the Emilia region; the seismic sequence was characterized by seven events with M_w higher than 5.0. The area struck by the earthquake was very large, including the provinces of Modena, Ferrara, Rovigo, and Mantova. Peak Ground Acceleration (PGA) registered at the closest station (epicentral distance equal to 16 km) during the mainshock was equal to 0.27g ([Chioccarelli et al, 2012](#)). Most of observed damage involved masonry buildings, precast industrial structures, and, in some cases, Reinforced Concrete (RC) structures; as it is documented by reconnaissance reports ([EPICentre Field Observation Report No. EPI-FO-200512, 2012](#); [EPICentre Field Observation Report No. EPI-FO-290512, 2012](#); [Parisi et al., 2012](#); [Liberatore et al., 2013](#), among others). The 20th of May mainshock was followed by another significant event on the 29th of May ($M_w=5.8$, according to INGV).

Most of the structures in the area struck by the earthquake are designed for gravity loads only. This observation is based on building stock's age of construction data of the Italian National Institute of Statistics (ISTAT) and considering the evolution of the seismic classification of Emilia region (see sections 2 and 3). RC buildings represent less than 20% of the whole building stock in the region and number of storeys data emphasizes that most of them are low/mid-rise buildings.

Photographic documentation on RC buildings showed that most of observed damage involved non-structural elements such as masonry infills. In few cases brittle failures in RC elements, either caused by local interaction with infills, or by poor reinforcement detailing, were the main cause of severe structural damage, see section 4.

Preliminarily, a code based seismic assessment is provided through a spectral based approach on bare RC buildings, leading to code-based bounds for spectral acceleration capacities to be compared with spectra of the records at the station closest to the epicentre of the mainshock event, see section 5. Results of code assessment on bare RC structures cannot be compared with observed damage data, given the key role played by masonry infills and not taken into account in such procedure.

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Hence, a damage assessment procedure (*FAST* method), on infilled RC buildings, is described and applied for the case of Emilia building stock. *FAST* damage assessment procedure belongs to the wider family of vulnerability assessment methodologies based on spectral displacements (e.g., NIBS 1997, 1998, 2002; Kircher et al., 1997; Erdik et al., 2004, among others). Damage states are classified according to the 1998 European Macroseismic Scale (Grunthal, 1998), *similarly to other vulnerability approaches available in literature in recent years* (e.g., Tertulliani et al. 2011; Acha and Adam 2012, among others). Results in terms of PGA capacities for damage states (DS) influenced by infills (DS1 up to DS3) are carried out, and compared with PGA recorded in the epicentral area during the 20th of May mainshock. Damage to infilled RC buildings, in the epicentral area, was considered as benchmark for *FAST* simplified damage assessment procedure, leading to a fair agreement between numerical and observed data, see section 6. *FAST* method was already employed for Lorca 2011, Spain, earthquake (De Luca et al., 2013a). Poor input data necessary for its employment and its simple application make it a suitable tool for damage mapping in the phase of emergency management right after seismic events (e.g., Goretti and Di Pasquale, 2006).

2. EMILIA BUILDING STOCK: ISTAT DATA

ISTAT (*Istituto Nazionale di Statistica*, Italian National Institute of Statistics) survey is a nation-wide census that provides information on citizens, buildings and dwellings. In particular, in the “14th general census of the population and dwellings” (14° *Censimento generale della popolazione e delle abitazioni*, ISTAT 2001) information about the number of storeys, as well as characteristic of residential buildings, and in some cases even those non residential, are provided. Collected information concerns category of use (industrial or residential), structural typology (masonry, reinforced concrete, ...), number of storeys, and age of construction. ISTAT data referred to the 448 Municipality hit by the 2012 earthquake are shown in Figure 1.

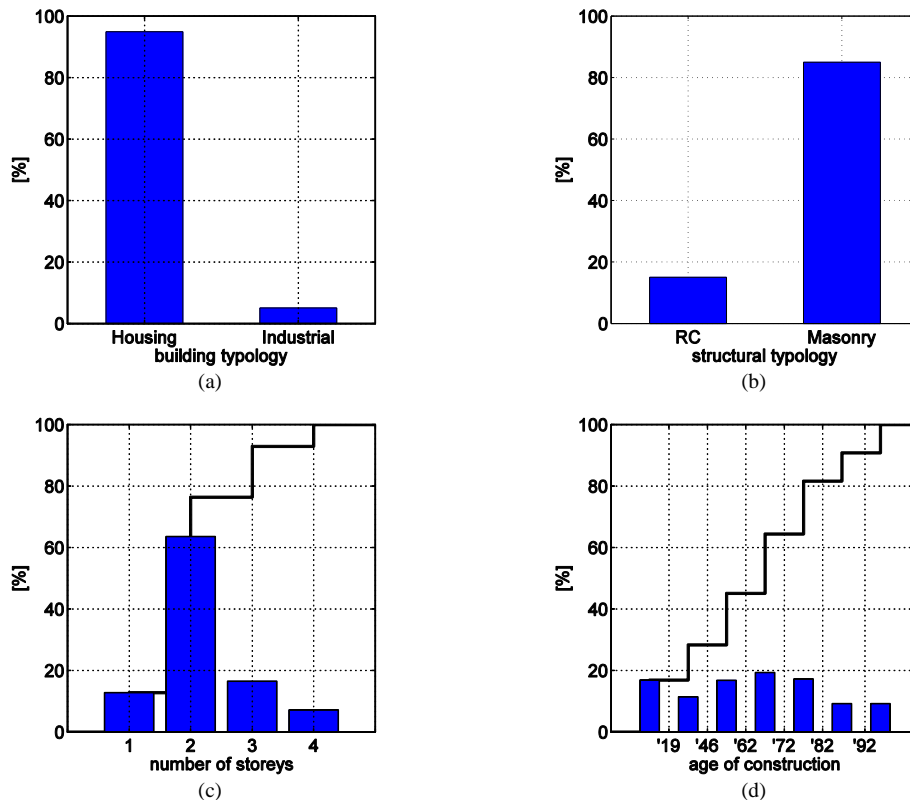


Figure 1. ISTAT statistics for the 448 Municipality hit by the 2012 Emilia earthquake: percentages with respect to building typology (a), building type (b), number of storeys (c), age of construction (d).

Data on age of construction of Emilia building stock need to be crossed with evolution of the seismic classification of the region in the last decades. This comparison allows identifying code

provisions according to which buildings have been designed, see section 3. Nevertheless, due to confidentiality requirements, these statistics are collected in aggregate manner; for example, it is not possible to get the number of RC buildings in a *census tract* (the spatial unit of ISTAT survey) dated back to a specific age of construction and characterized by a given number of storeys.

Emilia–Romagna is one of the richest, most developed regions in Europe. Bologna, its principal city, has one of Italy's highest quality of life indices and advanced social services. Data in [Figure 1a](#) show that the category of use of almost 5% of the whole building stock is constituted by buildings or group of buildings used as hotels, offices, commerce and industry, communications and transport. RC buildings represent less than 20% of the whole building stock, ([Figure 1b](#)). Storey distribution in [Figure 1c](#) emphasizes that two storey buildings represents about 60% of the whole building stock. Age of construction data show that most of building stock (more than 80%) have been realized before 1982, see [Figure 1d](#).

Data in [Figure 1](#) allow inferring that RC building stock is characterized mostly by low to medium rise buildings and the main part of them was realized between 1960 and 1980. The inferred information on the building stock can be compared to the corresponding information collected for L'Aquila (Abruzzo) area after the 2009 earthquake (e.g., [Ricci et al., 2011a](#)). The comparison of Emilia and Abruzzo building stock data emphasizes a similar number of RC building (approximately equal to 20%) and similar distribution of number of storeys and age of construction. On the other hand, given the different evolution of the seismic classification, similar building stock characteristics can lead to different design approaches, and to different structural performances.

3. EVOLUTION OF THE SEISMIC CLASSIFICATION IN THE EMILIA REGION

Seismic classification in Italy and, in general, in all seismically prone areas, is quite often a result of disastrous earthquakes. In Italy, the first classification was released in 1909, after the disastrous earthquake of Reggio Calabria and Messina in 1908. After this first classification every five or ten years a new update of seismic classification and code provisions were provided ([Lai et al., 2009](#)).

In recent years, four are the fundamental dates for the evolution of the seismic classification in Italy: 1984, 1998, 2003, and 2008. After Friuli (1976) and Irpinia (1980) disastrous earthquakes, three different seismic categories were set up according to De Marco and Marsan classification ([1986](#)). First, second, and third categories were characterized by a PGA equal to 0.10g, 0.07g, and 0.04g. Such accelerations were determined through the seismic coefficient S equal to 12, 9, and 6, and decreasing with the increasing of the category from first to third. According to the latter classification ([De Marco and Marsan, 1986](#)) most of the area struck by the 2012 Emilia earthquake was classified as non seismic (see [Figure 2a](#)). In 1998, a reclassification proposal was provided by *Servizio Sismico Nazionale*. According to 1998 classification the area struck by the 2012 earthquake was classified as seismic (third category) for the first time. This classification was never adopted officially by any code. After Molise (2002) earthquake, a significant revolution in Italian seismic design regulations and seismic classification was introduced through a new regulation document, OPCM 3274 ([2003](#)). OPCM 3274 introduced modern design rules (e.g., *capacity design*). It should be noted that OPCM 3274 was a recommendation document, and it was still possible to design new structures according to the previous obsolete code ([DM 16/01/1996](#)), in which PGA values were evaluated through S coefficients. On the other hand, local authority of Emilia region, in 2005, made the seismic classification of OPCM 3274 compulsory for the area. According to the classification of OPCM 3274, the whole area was still in third category; but the design PGA on rigid soil was equal to 0.15g. OPCM 3274 ([2003](#)) represented the “Copernican revolution” of Italian earthquake engineering, and it was very similar to the provisions provided in Eurocode 8 or EC8 ([CEN, 2004](#)). The last step in terms of seismic classification was made in 2008, when DM 14/01/2008 or NTC 2008 ([2008](#)) was released as regulation for design and assessment in Italy. In NTC 2008 ([DM 14/01/2008](#)), hazard data based on a 5km spaced grid are employed for determination of PGA, ([Stucchi et al., 2011](#)), see [Figure 2b](#). Spectral shape is site dependant, ending up in a code spectrum very close to a uniform hazard spectrum (UHS). It should be finally noted that the 2008 code became the official Italian code, and the only one to be employed, only in July 2009, after 2009 L'Aquila earthquake. Considering that most of the building stock was realized between sixties and eighties it is easy to recognize that most of the area struck by the 2012 Emilia earthquake was designed for gravity loads only.

In Figure 3a code spectra according to OPCM 3274 (2003), EC8 (2004), and NTC2008 (2008) are evaluated for epicentre coordinates of 20th of May event in the case of A (stiff) and D (soft) soil classes. The NTC 2008 spectrum depends on geographic coordinates of the site and soil class; it has a probability of exceedance of 10% in 50 years.

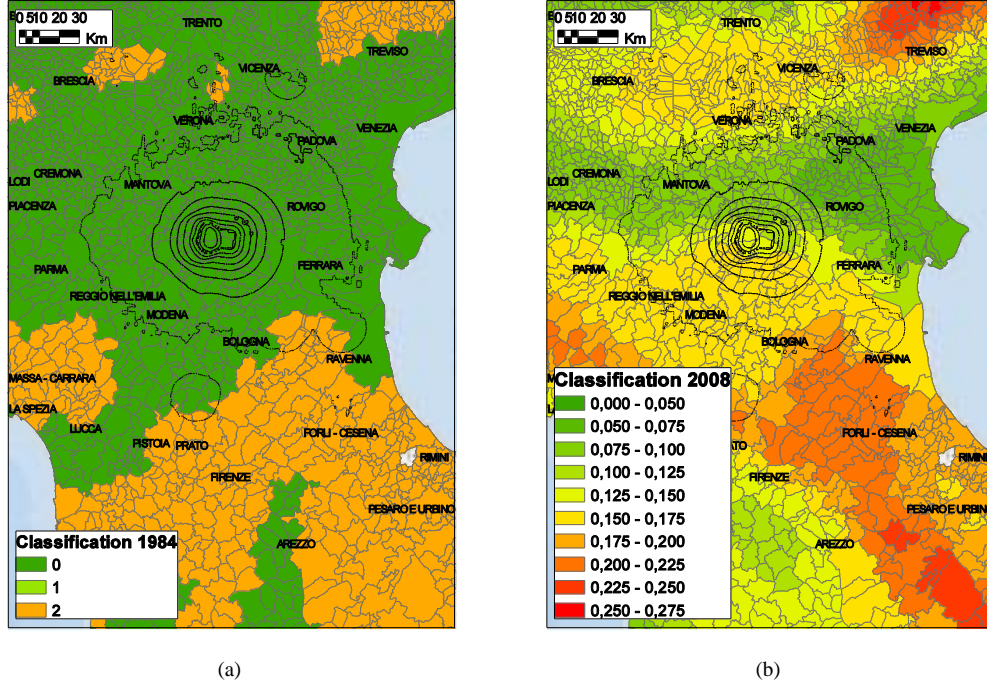


Figure 2. Seismic classification before 1998, according to De Marco and Marsan (1986), (a); and actual classification according to the official hazard data (Stucchi et al., 2011) employed in NTC 2008 (2008), (b), in both cases PGA contour lines of the shake map of the mainshock are shown.

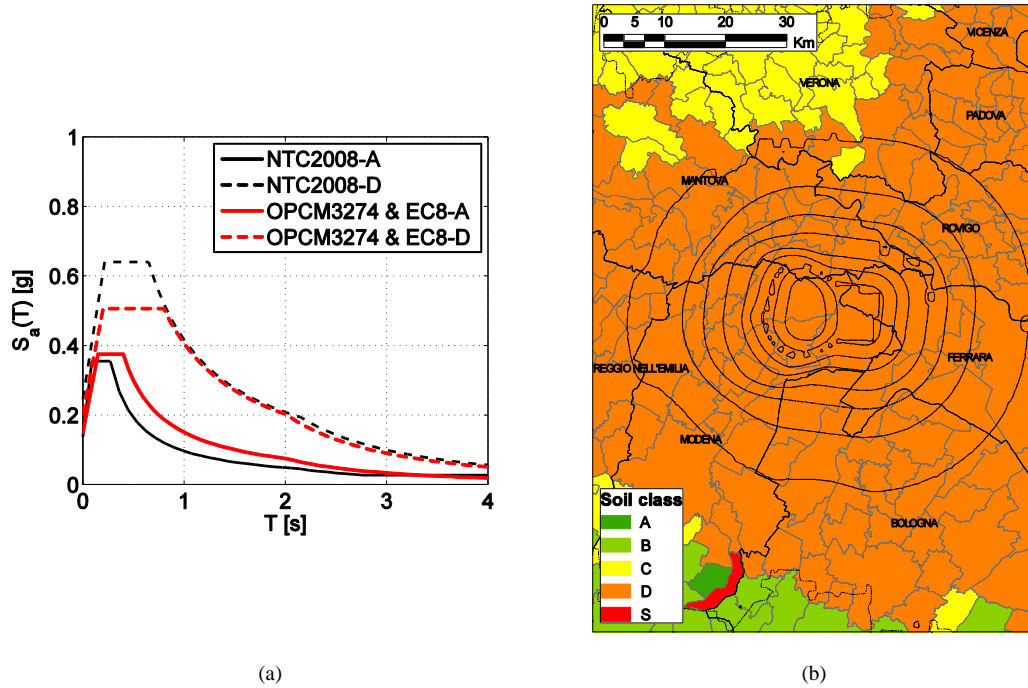


Figure 3. Comparison of code spectra according to OPCM 3274 (2003), EC8 (CEN, 2004) and NTC 2008 (DM/14/01/2008) for A and D soil classes at the epicenter of the event of the 20th of May (lat. 44.89, long. 11.23) (a); soil classification on geological basis of the area struck by the earthquake (Di Capua et al., 2011), and PGA contour lines of the shake map of the mainshock, (b).

OPCM 3274 and EC8 spectra depends on seismic category and soil class. Soil classes from A to E are classified according to the value of the average shear wave velocity in the top 30 meters of soil layers. OPCM 3274, EC8, and NTC 2008 share same soil classification. PGA on A soil class is equal to 0.138g according to NTC 2008, while it is equal to 0.15g in the case of OPCM and EC8. Soil amplification factor for D soil type is 1.8 in the case of NTC 2008 and 1.35 in the case of EC8 (type 1) and OPCM 3274.

Soil class D is the most frequent class in the whole epicentral area, see Figure 3b. Soil classification was taken from SEE-GeoForm project (Di Capua et al., 2011), which provides an open source WebGis toolbox giving geological, geomorphologic, geotechnical and geophysics data nationwide.

In the following all spectral based approaches are carried out considering EC8 spectra for D soil class, see Figure 3a. In fact, the area struck by the earthquake was very large and site-dependent characteristic of NTC 2008 spectra is not well suited for simplified large-scale approaches provided in the following.

4. EVENT AND DAMAGE

The Emilia 2012 sequence was characterized by seven events of moment magnitude (M_w)>5, five of which occurred between the 20th of May and the 29th May. These earthquakes caused structural and non-structural damage over a wide area. The events produced cumulative damage to structures (Iervolino et al., 2012). The M_w 6.0 mainshock occurred on the 20th of May 2012. In Figure 4 the shake map of the mainshock, provided by INGV, is shown.

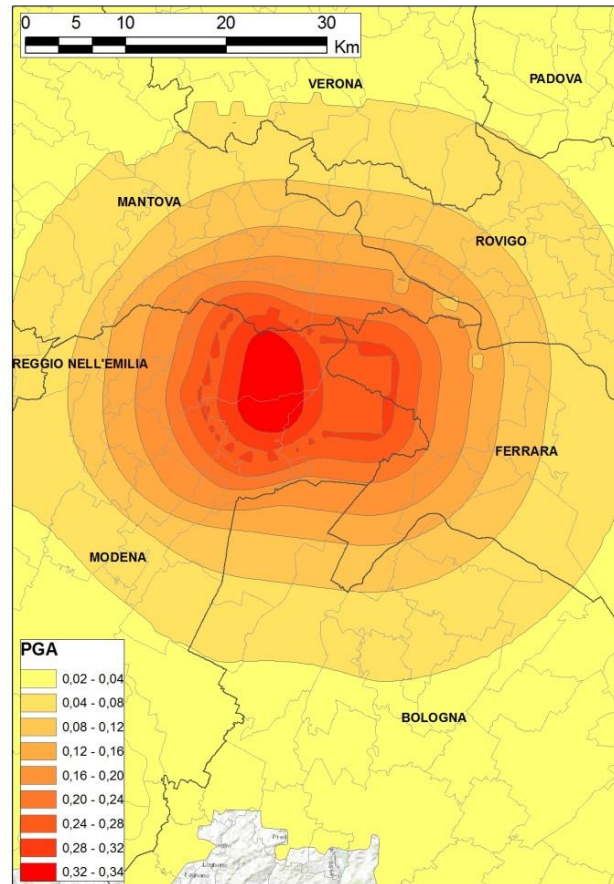


Figure 4. Shake map of the mainshock event occurred on the 20th of May 2012 and epicentre coordinates (lat 44.89; long. 11.23).

Observed damage was mostly registered on industrial precast buildings and masonry structures; damage to RC structures cannot be considered as representative of the sequence.

According to data collected during field surveys right after the events of the 20th and the 29th of May (e.g., [EPICentre Field Observation Report No. EPI-FO-200512, 2012](#); [EPICentre Field Observation Report No. EPI-FO-290512, 2012](#); [Parisi et al., 2012](#); [Liberatore et al., 2013](#)), it can be observed that RC buildings were characterized by slight or moderate damage and, seldom, structural collapses ([EPICentre Field Observation Report No. EPI-FO-290512, 2012](#)) were observed.

A preliminary analysis of structural typologies and observed damage represents the first step towards a quantitative study of RC buildings performance during the 2012 Emilia earthquake. Most of observed damage affected non-structural elements. Rarely structural elements showed significant damage, and in most cases brittle failures occurred. Most of damage to RC structures was observed in the area close to the epicentres of the events of the 20th of May and the 29th of May, close to the towns of Mirandola, Cavezzo, and San Felice sul Panaro. In [Figures 5 to 9](#), an overview of structural and non-structural damage is collected. In [Figures 5 and 6](#), two four- and five-storey RC buildings are shown; both buildings are located in the centre of Mirandola. Damage shown in [Figure 5](#) can be classified as slight; in fact, in both cases only the plaster of the masonry infills is cracked. Such cases can be classified as the situations in which post event retrofitting interventions are very limited. Buildings in [Figure 5](#) are an example of good seismic performances, notwithstanding the fact that they were likely designed for gravity loads only. In [Figure 6](#), an example of moderate damage to infills of an RC building in Mirandola is shown. Non-structural damage involved the external layer of the infill panels. In [Figure 6b](#) damage to the external layer allows the visual inspection of structural elements that did not show any damage.

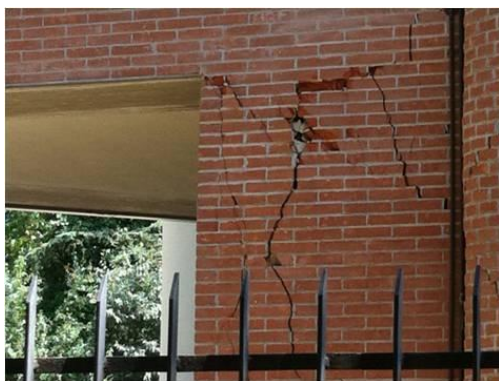


(a)



(b)

Figure 5. Mirandola, slight damage to plaster on masonry infill panels; ([Parisi et al., 2012](#)).



(a)



(b)

Figure 6. Mirandola, moderate damage to the external layer of brick infills emphasizing the absence of damage to RC elements of the building; ([Parisi et al., 2012](#)).

[Figure 7](#) shows a seven-storey building in which both structural and non-structural damage is observed. Damage to the external layer of infills, characterized by the crushing of the corners of the panels is shown in [Figure 7a](#). Such damage seems a typical *corner crushing* failure of the infills. In the cropped image of [Figure 7a](#), shown in [Figure 7b](#), the diagonal cracking of the head of

the columns adjacent to the damaged panel is clearly shown. 45° slope of the crack in Figure 7b suggests a brittle failure in the columns likely induced, on one hand, by the local interaction with the infill panels, and, on the other hand, by the poor local detailing of the element.

In Figure 7c and Figure 7d typical structural damage due to local interaction with infill panels is shown. In Figure 7c, a partial infill results in a reduced shear span ratio of the adjacent columns (*squat columns*); it is well known that these kinds of structural situations favour the occurrence of brittle failures (De Luca and Verderame, 2013). In fact, the reduced shear span ratio leads to increasing shear demand (at most equal to the minimum between the peak shear induced by the infills and the plastic shear of the squat column). In Figure 7d, low percentage of transversal reinforcement (about 20 cm spaced stirrups) is not enough to guarantee proper shear capacity for the shear demand increased by the presence of partial infills. In fact, 20 cm stirrup spacing is very poor if compared to actual prescriptions provided by Italian and European seismic codes (DM 14/01/2008; CEN, 2004) in these cases.



Figure 7. Mirandola, (a) damage to the external layer of the infill panels at the first storey of the building, (b) diagonal cracking characterized by shear failure at the head of the RC column; structural damage in a squat RC columns because of the partial infilling as a result of the local interaction between infill and RC elements, (c) view of the façade of the building, (d) zoom-in of the squat column; (e) moderate damage and diagonal cracking of the infill panels in the between of two openings, (f) significant damage and partial collapse of an infill panel; (Parisi et al., 2012).

In [Figure 7e](#) and [Figure 7f](#) two examples of moderate and significant damage to infill panels are shown, respectively. In [Figure 7e](#), diagonal cracking of an infill panel between two openings is shown. Such kind of damage is very similar to the typical failure mode of structural masonry panels in masonry structures. The situation of a partially collapsed infill in [Figure 7f](#) recalls the typical collapse mechanism of sliding shear, characterized by cracking in the middle of the panel and consequent out-of-plane failure of the top part of the infill ([Shing and Mehrabi, 2002](#)).

The seven-storey building in Mirandola shown in [Figure 7](#) is characterized by significantly different seismic performances with respect to the buildings in [Figure 5](#) and [Figure 6](#), characterized by four and five storeys, respectively. The three buildings in [Figures 5](#) to [7](#) are located in the same neighbourhood of Mirandola, as it can be seen from data in [Parisi et al. \(2012\)](#). Hence it can be reasonably assumed that there were no significant differences in local amplification of seismic action for the buildings, and they were designed and realized according to the same regulations.

Different performances of the three buildings can be related to different heights (e.g., number of storeys), and consequently to their different fundamental periods, in analogy with findings of other vulnerability studies available in literature (e.g., [Giovinazzi and Lagommarino, 2006](#); [Borzi et al., 2008](#); [Ricci, 2010](#)). On the other hand, the latter observation, based on three buildings, is not enough to represent a general trend.



Figure 8. San Felice sul Panaro, significant damage in RC elements because of local interaction with infill panels ([Liberatore et al., 2013](#)).



Figure 9. Cavezzo, (a) structural collapse of a RC building infilled with hollow clay bricks, (a) zoom-in of a beam-column joint characterized by poor seismic detailing; ([EPI-FO-290512, 2012](#)).

Figure 8 shows the first storey of a three-storey building in San Felice sul Panaro characterized by an irregular distribution of the infill panels in height and plan (Liberatore et al. 2013). Views of the structure in Figures 8a and 8b suggests a collapse mode induced by the local interaction between infills and RC elements. Such kind of collapse, as it was already emphasized by Liberatore et al. (2013), is similar to damage observed after the 2009 L'Aquila earthquake (e.g., Verderame et al. 2011). The local interaction between infills and structural elements has increased the shear demand and, at the same time, poor seismic detailing of transversal reinforcement has contributed to an inadequate capacity against brittle failures, in both cases of conventional shear failures and sliding shear failures in the RC elements.

Finally, Figure 9 shows one of the very few examples of structural collapses that involved RC frame buildings after the 2012 Emilia sequence (EPI-FO-290512 2012). Even if the state of the building (see Figure 9a) does not allow any inference on the likely causes of the collapse, the detail, in Figure 9b, emphasizes absence of stirrups in the beam-column joints.

5. CODE BASED SEISMIC ASSESSMENT

Information collected in Section 2 and 3 allow a characterization of building stock in terms of number of storeys and design approach. A preliminary code-based seismic assessment is carried out through this information. The procedure provided herein is based on a spectral based approach similar to that employed in HAZUS (NIBS, 1997, 1999, 2002).

In Figure 10a, code spectra evaluated at Mirandola station (MRN, lat 44.8782, long 11.0617) for D soil class in the case of OPCM 3274 (2003), EC8 (CEN, 2004), and NTC2008 (DM14/01/2008) are compared to the horizontal components registered at the site. MRN station is the closest station (epicentral distance equal to 16km) that registered the mainshock on the 20th of May 2012 (Chioccarelli et al., 2012). Figure 10a emphasizes how the spectrum provided by actual Italian seismic code (DM14/01/2008), characterized by a return period of 475 years, is comparable to the registered components during the mainshock event (Iervolino et al., 2012).

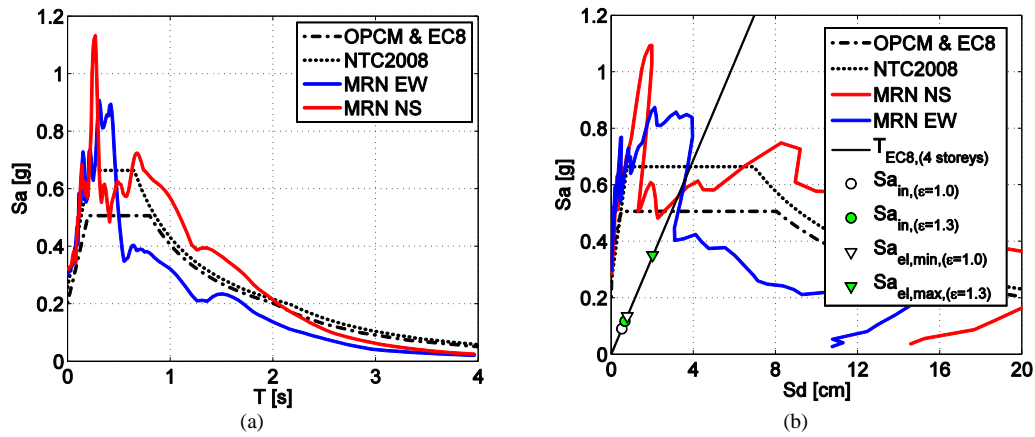


Figure 10. Acceleration spectra at Mirandola site (lat 44.8782, long 11.0617) according to OPCM 3274 (2003), EC8 (CEN, 2004), and NTC2008 (DM14/01/2008) on D soil class, compared to the horizontal component registered on the 20th of May 2012 at MRN station, (a); and corresponding ADRS spectra compared with code-based seismic performances of bare (four storeys) RC buildings, (b).

According to the evolution of the seismic classification, it was shown that most of RC building stock was designed for gravity loads only and realized between 60s and 80s. The seismic capacity of a gravity load designed RC building is assumed equal to that of a structure designed for the lowest design acceleration provided by obsolete seismic design provisions (DM 3/3/1975; DM16/01/1996). In particular, according to the 1998 classification, the area struck by the earthquake was classified in third seismic category.

The value of the seismic coefficient (S) for third category was equal to 6. Seismic coefficient S was necessary for the evaluation of seismic intensity, C , see equation 1. The inelastic design spectra, $S_{a,d}(T)$, was then defined through four other parameters, see equation 2. R was the so called response coefficient, assumed as a function of the fundamental period (T), while coefficients ϵ and β respectively depended on soil compressibility ($\epsilon=1.0-1.3$) and structural typology, $\beta=1.0$

for moment resisting frame (MRF), and $\beta=1.2$ for shear wall (SW) structures. Finally, I was the importance factor of the building according to its category of use, see Ricci et al. (2011a) for further details.

$$C = (S - 2)/100 = (6 - 2)/100 = 0.04 \quad (1)$$

$$S_{a,d}(T)/g = C \cdot R \cdot \varepsilon \cdot \beta \cdot I \quad \begin{cases} R = 1.0 & \text{for } T \leq 0.8s \\ R = 0.862/T^{2/3} & \text{for } T > 0.8s \end{cases} \quad (2)$$

According to equations 1 and 2, it can be assumed, for low-mid rise (see Figure 1c) RC MRF buildings, a fundamental period lower than 0.8s and, consequently, $R=1.0$, $\beta=1.0$ and $I=1.0$. Considering that soil class D is dominant in the area, ε factor should be considered equal to 1.3. On the other hand, it is not sure that compressibility of the soil was properly considered at the age of construction. Hence, in the following, both the cases of ε equal to 1.0 and 1.3 are going to be considered. In the case of $\varepsilon=1.0$, $S_{a,d}(T)$ is equal to 0.04g, while for $\varepsilon=1.3$, $S_{a,d}(T)$ is equal to 0.052g. On the other hand, once in 1996 limit state design was firstly introduced in Italy as an alternative to allowable working stress approach (Coignet and De Tedesco, 1894; Park and Paulay, 1975) as a possible design alternative, the spectral acceleration was characterized by 50% amplification. Finally, in the case of limit state design, the seismic capacity for low-medium rise building can be assumed equal to 0.06g and 0.078g, for $\varepsilon=1.0$ and $\varepsilon=1.3$, respectively.

It should be noted that in every design procedure (in both cases of allowable working stress and limit state approaches) design material strengths are derived from characteristic strengths, while building performance in the assessment framework should be evaluated considering mean strength. In particular, the ratio between mean and design strength values represents an overstrength contribution to the minimum inelastic acceleration capacity of existing buildings. According to previous studies available in literature (e.g., Borzi and Elnashai, 2000; Galasso et al., 2011), the so called material overstrength factor can be assumed equal to 1.50, finally leading to the evaluation of the *inelastic acceleration capacity* ($S_{a,in}$). In the case of low-medium rise buildings designed in the Emilia region $S_{a,in}$ can be assumed equal to 0.09g ($=0.06g \times 1.50$) and 0.117g ($=0.078g \times 1.50$).

Assuming the extreme values of the behaviour factor range suggested by the Italian seismic regulation for existing buildings (Circolare 617, 02/02/2009), respectively equal to 1.50 and 3.00, it is possible to obtain the minimum and the maximum *elastic spectral acceleration capacities* ($S_{a,el,min}$, $S_{a,el,max}$) according to a code-based framework, multiplying the inelastic acceleration and the behaviour factor. For the specific $S_{a,in}$ considered above, and equal to 0.09g and 0.117g, $S_{a,el,min,(\varepsilon=1.0)}=0.135g$ and $S_{a,el,max,(\varepsilon=1.0)}=0.27g$, while for $\varepsilon=1.3$, $S_{a,el,min,(\varepsilon=1.3)}=0.175g$ and $S_{a,el,max,(\varepsilon=1.3)}=0.35g$, respectively.

In Figure 10b, the inelastic spectral acceleration values and lower and upper bounds of elastic acceleration values ($S_{a,el,min,(\varepsilon=1.0)}$ and $S_{a,el,max,(\varepsilon=1.3)}$) are represented in the acceleration displacement response spectra (ADRS) format along the slope characterizing the fundamental period (T_{EC8}) of four-storey buildings. T_{EC8} is evaluated according to Eurocode 8 formulation and equal to 0.48s; the interstorey height (3.0m) and the number of storeys (4) have been selected considering frequent characteristics of RC buildings in the area of Mirandola. $S_{a,el,min,(\varepsilon=1.0)}$ and $S_{a,el,max,(\varepsilon=1.3)}$ are compared with code ADRS for EC8, OPCM, and NTC2008. Finally, $S_{a,el,min,(\varepsilon=1.0)}$ and $S_{a,el,max,(\varepsilon=1.3)}$ are compared to the ADRS of the two horizontal components registered at MRN station on the 20th of May. Comparisons in Figure 10b emphasize that code based seismic performances are strictly lower than registered demand and that evaluated according to codes.

Notwithstanding the fact that code-based approaches still represent a lower boundary for seismic capacity, observed damage, shown in the above section, suggest that such performance estimation for bare structures is very conservative and it cannot be considered a benchmark for the comparison with data from field surveys (excluding the occurrence of brittle failures). In fact, a code-based assessment cannot account for masonry infill structural contribution (Gómez-Martínez et al., 2012; Ricci et al. 2013).

6. DAMAGE ASSESSMENT OF INFILLED RC BUILDINGS

A simplified damage assessment procedure for infilled RC MRF buildings is described, accounting for the presence of infills. *FAST vulnerability approach* allows a preliminary damage assessment thanks to:

- (i) a simplified evaluation of a capacity curve of an infilled RC building (Gómez-Martínez et al., 2012);
- (ii) an approximate mechanical interpretation of Damage States according to EMS-98 scale (Grunthal, 1998);
- (iii) evaluation of seismic capacity through an intensity measure (elastic spectral acceleration or PGA) defining the IN2 curve (Dolšek and Fajfar, 2005) corresponding to the capacity curve defined in (i).

FAST approach is a rapid large-scale assessment method (Verderame et al., 2012a; Gómez-Martínez et al., 2012; De Luca et al., 2013a). The application of such methodology asks for very basic information and data on the building stock of the area to be studied: number of storeys, age of construction, design code (e.g., according to the evolution of the seismic classification), typical structural and nonstructural material properties at the time of construction (e.g., Verderame et al., 2001; Verderame et al., 2012b; Circolare 617, 02/02/2009). *FAST* approach has been validated through detailed damage assessment, showing a fair agreement with observed damage data (Verderame et al., 2012a).

6.1 Simplified evaluation of capacity curve of infilled RC buildings

It is well known that structural contribution of infills provides an increase in lateral strength and stiffness of the building (resulting in a decrease in fundamental period), but also leads to a global strength degradation after the attainment of the maximum resistance, given their brittle behavior (see Figure 11a). The structural behavior described above is mainly representative of uniformly infilled existing buildings (Dolšek and Fajfar, 2001, Ricci et al. 2013).

The simplified capacity curve of a uniformly infilled RC MRF building can be represented by a quadrilinear backbone (Dolšek and Fajfar, 2005) characterized by an initial elastic plastic backbone (with the plateau at the maximum base shear strength V_{max}), followed by a softening branch up to the minimum base shear strength (V_{min}). According to *FAST* approach, the simplified capacity curve has a softening branch characterized by a drop, (see the dashed line in Figure 11b). The latter is an additional simplifying hypothesis with respect to the idealized backbone provided by Dolšek and Fajfar (2005), and represents the limit situation in which softening slope is infinite (significant brittle behavior of infills). Even if more accurate piecewise linear approximations of the capacity curve are available in literature (De Luca et al. 2013b), Dolšek and Fajfar's quadrilinear idealization is made for the R - μ - T relationship that is going to be employed; so it is well suited for the framework of this study (see section 6.2).

Figure 11 shows the ideal steps of *FAST* approach leading to the definition of the capacity curve in ADRS format (see Figure 11c). In Figure 11a, the typical shape of a pushover curve on infilled RC structures is shown with a qualitative example of the contribution provided by infills and RC frames. The idealized capacity curve is shown in Figure 11b and 11c, respectively in base shear-top displacement format and ADRS format.

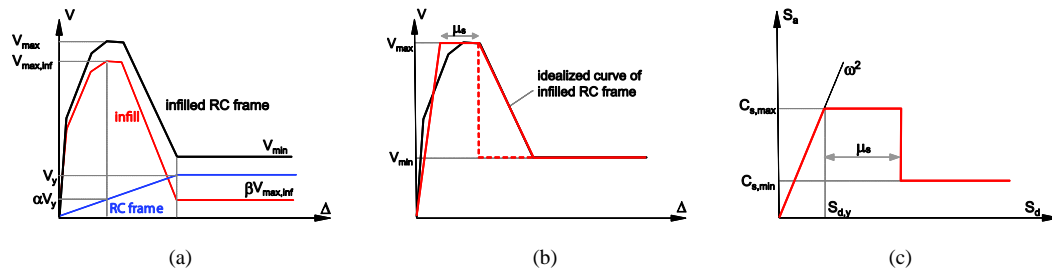


Figure 11. Example of infilled RC frame pushover curve (a), its quadrilinear idealization in base shear-displacement format (V - Δ) in which the dashed line indicates the approximation introduced in *FAST* approach (b), the capacity curve in the ADRS format (c).

The simplified capacity curve of infilled RC structures asks for the definition of four characteristic parameters. According to the representation in [Figure 11c](#), the capacity curve in ADRS format can be defined through the definition of four parameters:

- $C_{s,max}$, the inelastic acceleration of the equivalent single degree of freedom (SDOF) at the attainment of the maximum strength V_{max} ;
- $C_{s,min}$, the inelastic acceleration of the equivalent SDOF at the attainment of the plastic mechanism of the RC structure (all the infills of the storeys involved in the mechanism have attained their residual strength $\beta V_{max,inf}$);
- μ_s , the available ductility up to the beginning of the degradation of the infills;
- T , the equivalent elastic period computed from the fundamental period T_o of the infilled RC building.

[Equations 3](#) and [4](#) show the formulations assumed for the definition of the first two parameters. The value of μ_s was assumed equal to 2.5. This assumption was made through the comparison of detailed assessment studies available in literature on gravity load designed buildings ([Ricci, 2010](#); [Verderame et al., 2012a](#)).

$$C_{s,max} = \frac{V_{max}}{\Gamma_1 m_1} = \frac{\alpha V_y + V_{max,inf}}{\Gamma_1 m_1} = \frac{\alpha V_y + \tau_{max} \cdot \rho_{inf} \cdot A_b}{\Gamma_1 m_1} = \alpha C_s + \frac{\tau_{max} \cdot \rho_{inf} \cdot A_b}{(N \cdot m \cdot A_b \cdot \lambda)} = \alpha C_s + \frac{\tau_{max} \cdot \rho_{inf}}{N \cdot m \cdot \lambda} \quad (3)$$

$$C_{s,min} = \frac{V_{min}}{\Gamma_1 m_1} = \frac{V_y + \beta V_{max,inf}}{\Gamma_1 m_1} = \frac{V_y + \beta \tau_{max} \cdot \rho_{inf} \cdot A_b}{\Gamma_1 m_1} = C_s + \beta \frac{\tau_{max} \cdot \rho_{inf} \cdot A_b}{(N \cdot m \cdot A_b \cdot \lambda)} = C_s + \beta \frac{\tau_{max} \cdot \rho_{inf}}{N \cdot m \cdot \lambda} \quad (4)$$

where:

- $V_{max,inf}$ is the maximum base shear provided by the infills, see [Figure 11a](#);
- V_{min} is the base shear at the attainment of the plastic mechanism of the RC structure, see [Figure 11a](#);
- Γ_1 is the first mode participation factor;
- m_1 is the mass of the equivalent SDOF (the generalized mass corresponding to the first vibration mode);
- N is the number of storeys;
- m is the average mass of each storey normalized by the building area (e.g., equal to 0.8t/m² for residential buildings);
- λ is a coefficient for the evaluation of the first mode participant mass respect to the total mass of the multiple degree of freedom, MDOF, equal to 0.85 for buildings with more than three storeys ([CEN, 2004](#));
- τ_{max} is the maximum shear stress of the infills according to Fardis ([1997](#));
- ρ_{inf} is the ratio between the infill area (evaluated along one of the principal directions of the building) and the building area A_b ;
- α, β are coefficients that account, respectively, for RC elements' strength contribution at the attainment of $V_{max,inf}$ and for the residual strength contribution of the infills at the attainment of the plastic mechanism of the RC structure, see [Figure 11a](#).

[Equations 3](#) and [4](#) are evaluated in the hypothesis of the attainment of a soft storey plastic mechanism of the structures at the first storey. In fact, in the case of existing buildings (not capacity designed), column-sway storey collapse mechanisms are likely to occur, especially if infills are present, due to their relatively high strength contribution compared with RC structural elements. In particular, for uniformly infilled RC buildings, the strength contribution of infill elements leads to an approximately uniform distribution of storey shear strength along the height of the building; thus, damage is concentrated at the first storey, where the shear demand is higher ([Dolce et al., 2005](#)). The subsequent drop in strength, caused by the brittle nature of infills, finally leads to the development of a soft storey ductile mechanism, with a residual strength mainly provided by RC columns at ground level.

The last parameter to be evaluated for the definition of the capacity curve is the equivalent elastic period T , defined in analogy with the original approach by Dolšek and Fajfar ([2005](#)). The first branch of the curve ([Figure 11b](#)) represents both the initial elastic and the post-cracking

behaviour occurred in both RC frame and infills. Hence, the equivalent elastic period T , in the idealized capacity curve (Figure 11c), is higher than the fundamental elastic period T_0 , correspondent to the tangent stiffness at the beginning of the capacity curve. In particular, in this study, T is evaluated from a relationship with T_0 .

The fundamental period T_0 of an infilled RC MRF can be obtained from the formulations available in literature obtained from regressions on experimental data obtained by means of techniques based on *low amplitude motion*, as microtremors or ambient noise. Several of these studies (e.g., Oliveira and Navarro, 2010 among others) were carried out on infilled RC buildings in Mediterranean area. Due to the nature of the applied measurement techniques, it is likely to assume that these relationships provide a value of the period corresponding to a linear behaviour of structural materials and components. All the studies mentioned above lead to period formulation ranging in the interval in equation 5.

$$T_0 = (0.042 \div 0.054)N \quad (5)$$

Notwithstanding the fact that Oliveira and Navarro (2010) emphasized how experimental and numerical data on fundamental period of infilled RC buildings are quite different, the numerical study provided in Ricci et al. (2011b) shows a very good agreement with the same data. In particular, Ricci et al. (2011b) provided different formulations, see equations 6.

$$T_0 = 0.049N \quad (6a)$$

$$T_0 = 0.016H \quad (6b)$$

$$T_0 = 0.023 \frac{H}{\sqrt{100\rho_{\text{inf}}}} = 0.0023 \frac{H}{\sqrt{\rho_{\text{inf}}}} \quad (6c)$$

Equation 6a confirms the good agreement with equation 5. Equation 6b is obtained from equation 6a considering a constant interstorey height equal to 3.0m; while equation 6c is characterized by a closer agreement with numerical data (Ricci et al. 2011b). In FAST approach, equation 6c is employed, given its good agreement with experimental data and the presence of variables already employed in the evaluation of the simplified capacity curve, see equations 3 and 4. Finally, the switch from T_0 to the equivalent elastic period T is made through the amplification coefficient κ , calibrated on detailed analytical data (Manfredi et al., 2012; Verderame et al., 2012a). The evaluation of the lateral strength of the bare structure (V_y), see Figure 11a, was made according to the simulated design procedure described in the next section.

6.1.1 Simplified simulated design procedure

FAST approach is based on a simplified procedure for the definition of the capacity curve of existing buildings both in the case of obsolete seismic design and gravity load design. In the first case the lateral force of the bare structure can be defined by means of the design spectral acceleration employed at the time of construction (Gómez-Martínez et al., 2012; De Luca et al 2013a). In the case of design for gravity loads only, it is necessary to employ a simulated design procedure. The simulated design shown herein should be considered as a simplified procedure derived from the detailed approach described in (Verderame et al., 2010).

Given the area in plan of the building (A_b), the number of storeys (N), dead loads (g), and live loads (q) per square meter, every storey will be characterized by a gravity load $p=(g+q)A_b$; while the whole building weight will be equal to $P=\sum_{i=1}^n (g+q)A_b$.

Given an average dimension of bays' length (l), it is possible to define the average tributary area of columns, specializing such evaluation to their position in plan (central, lateral, or corner). Once the tributary area of the central column $A_{tr}^j=l^2$ is defined, for lateral and corner columns the value will be equal to 50% and 25% of A_{tr}^j , respectively.

The previous evaluation allows defining the axial load $N_{c,j}^i$ on the j^{th} column at the i^{th} storey according to equation 7, in which the tributary area coefficient α_1 is equal to 1.00, 0.50, and 0.25

in the case of central, lateral, and corner columns, respectively. The section area of the columns, $A_{c,i}^j$, (making the simplified hypothesis of square sections, with width and height equal to a) can be evaluated as a function of the concrete allowable stress σ_c as shown in [equation 8](#).

$$N_{c,i}^j = \sum_i^n (g + q) \cdot \alpha_i \cdot A_{c,i}^j = \sum_i^n (g + q) \cdot \alpha_i \cdot l^2 \quad (7)$$

$$A_{c,i}^j = a^2 = \frac{N_{c,i}^j}{\sigma_c} = \frac{\sum_i^n (g + q) \cdot \alpha_i \cdot l^2}{\sigma_c} \quad (8)$$

The longitudinal reinforcement, in turn, can be computed considering minimum code prescriptions ([R.D. 2229/1939](#)) or typical construction practice; the latter can be expressed as a percentage of the minimum area necessary for the square section (a^2), in the following referred to as ρ_l . The flexural strength of such columns ($M_{Rc,i}^j$) can then be defined according to [equation 9](#), based on a simplified application of first principles. Coefficient β_l accounts for reinforcement distribution in the section (e.g. equal to 0.50 in the simplest case of only two registers); k accounts, in a simplified hypothesis, for the distance between the registers (e.g. equal to 0.80 in the simplest case of only two registers). Average compressive strength of the concrete and average steel yield strength are indicated with f_c and f_y , respectively. Given the flexural strength of the generic column, it is possible to determine its plastic shear at the j^{th} storey ($V_{plc,i}^j$) according to [equation 10](#), in which L_v is the shear span length, taken as one half of the interstorey height. For each storey, plastic shear is evaluated as the sum of the plastic shears of each column at the considered storey. The lateral strength of the bare structure can be defined as the plastic shear of the first storey (V_y), according to the hypothesis discussed in the previous section (first storey plastic mechanism).

$$M_{Rc,i}^j = \frac{N_{c,i}^j \cdot a}{2} \cdot \left(1 - \frac{N_{c,i}^j}{f_c \cdot a^2}\right) + \beta_l \cdot [\rho_l \cdot (a^2)] \cdot f_y \cdot (k \cdot a) \quad (9)$$

$$V_{plc,i}^j = \frac{M_{Rc,i}^j}{L_v} = \frac{1}{L_v} \left[\frac{N_{c,i}^j \cdot a}{2} \cdot \left(1 - \frac{N_{c,i}^j}{f_c \cdot a^2}\right) + \beta_l \cdot [\rho_l \cdot (a^2)] \cdot f_y \cdot (k \cdot a) \right] \quad (10)$$

This simplified simulated design procedure is based on allowable working stress design approach ([Coignet and De Tedesco, 1894](#); [Park and Paulay, 1975](#)) employed in Italy up to 1996 ([DM16/01/1996](#)). Allowable stress design approach for reinforced concrete was based on elastic theory, and it was thought that the straight line distribution of stress led to mathematical simplification. Allowable working stresses were typically chosen through reduction factors of concrete cubic compressive strength and steel yield strength ([R.D. 2229/1939](#)); further details on obsolete design practice can be found in Verderame et al. (2010).

6.2 Seismic capacity assessment

Starting from the simplified capacity curve, the corresponding IN2 curve can be obtained ([Dolšek and Fajfar, 2005](#)). The IN2 curve, in analogy with an IDA curve, is a relationship between an engineering demand parameter (EDP), e.g. SDOF displacement S_d , and an intensity measure (IM), e.g. elastic spectral acceleration $S_a(T)$ or PGA. When the IN2 curve in terms of SDOF S_d - $S_a(T)$ is considered, $S_a(T)$ corresponding to a certain value of displacement represents an $S_a(T)$ -capacity of the structure given that displacement. So, the *seismic capacity* expressed in terms of $S_a(T)$ is defined as the elastic spectral acceleration $S_a(T)$ under which S_d is equal to the *displacement capacity* of the structure. In the same way, seismic capacity expressed in term of PGA can be defined.

Values of $S_a(T)$ corresponding to characteristic values of the SDOF displacement (ductility), including the considered Damage States (DSs) analyzed in the following, are calculated, based on the R - μ - T relationships given in Dolšek and Fajfar (2004) for degrading response. Moreover, the assumed hypothesis on the drop of the capacity curve does not affect the evaluation of the corresponding IN2, since this parameter is discarded in the regression made for the determination of the R - μ - T relationship ([Dolšek and Fajfar, 2004](#)).

The evaluation of the SDOF displacement corresponding to a specified DS level ($S_{d/DSi}$) can be made as function of the interstorey drift ratio (IDR) at which the specific DS is attained. In fact, once the IDR_{DSi} is computed, the roof displacement Δ_{DSi} of the MDOF can be defined through the evaluation of the deformed shape. SDOF displacement $S_{d/DSi}$ is obtained from roof displacement Δ_{DSi} through the first mode participation factor Γ_1 evaluated in a simplified manner according to the tabulated values of the so called coefficient C_0 (equivalent to Γ_1) in (ASCE/SEI 41-06, 2007).

DSs in terms of IDR can be defined through the empirical-mechanical interpretation of the damage classification of the EMS-98 scale (Grunthal, 1998). Definition of IDR_{DSi} is made for the only DSs characterized by a specific infill damage level. In particular such procedure can be pursued up to DS3:

- **DS1: *fine cracks in partitions and infills*.** This DS is defined by the end of the phase in which infills are characterized by an elastic uncracked stiffness. The IDR_{DS1} could be evaluated as the drift characterizing the attainment of the cracking shear in the infill backbone (Fardis, 1997). Hence, from a mechanical point of view the lateral drift corresponding to this DS can be defined as the ratio of tangential cracking stress (τ_{cr}) to the shear elastic modulus (G_w) of the infills. Assuming the values suggested for these parameters in (Circolare 617, 02/02//2009), this ratio, for typical clay hollow bricks employed in Italy, is on average equal to 0.026%. This value is also similar to that proposed by Colangelo (2012) on the basis of experimental results in Colangelo (2003). According to the above observations, IDR_{DS1} is assumed equal to 0.03%.
- **DS2: *cracks in partition and infill walls, fall of brittle cladding and plaster*.** After cracking, with increasing displacement, a concentration of stresses along the diagonal of the infill panel takes place, together with an extensive diagonal cracking, up to the attainment of the maximum resistance. DS2 can be therefore assumed as corresponding to the achievement of maximum resistance. In a pure mechanical approach IDR_{DS2} could be evaluated as the drift corresponding to the peak of the backbone according to Fardis' model (1997). The secant stiffness to this point can be computed according to Mainstone's formulation (1971). Based on literature results (e.g., Ricci, 2010), the secant stiffness can be considered approximately as the 25% of the elastic one. Hence, considering (i) the assumption made for IDR_{DS1} , and (ii) the average ratio assumed by Fardis (1997) between peak and cracking tangential stress ($\tau_{max}/\tau_{cr}=1.3$), IDR_{DS2} could be assumed equal to 0.16% ($=0.03\% \times 4 \times 1.30$). This value is similar to that assumed in numerical-experimental investigation by Dolšek and Fajfar (2008), but significantly lower than that evaluated in Colangelo (2012) equal to 0.4%. Finally, the attainment of IDR_{DS2} was assumed at 0.20%.
- **DS3: *large cracks in partition and infill walls, failure of individual infill panels*.** At this stage the generic infill panel shows a significant strength drop with a consequent likely collapse. According to Fardis' backbone, the drift at this stage is strictly dependent on the softening stiffness of the infill. On the other hand the softening stiffness is characterized by a large variability depending on the specific kind of infill (mechanical properties, type of bricks, etc.). Dolšek and Fajfar (2008) suggested an IDR value for this stage five times the IDR at the attainment of peak strength, equal to 0.8% ($=0.16\% \times 5$). On the other hand Colangelo (2012) provides for the same stage ("many bricks have been broken") an IDR equal to 1.60%. Given the significant difference between the two evaluations, an intermediate evaluation is provided herein, and IDR_{DS3} is assumed equal to 1.20%.

Once the characteristic IDR_{DSi} are defined, roof displacement Δ_{DSi} , corresponding to a given damage state (DS_i), can be evaluated through a simplified evaluation of the deformed shape. In particular, the deformed shape at a given DS level is evaluated according to the two following assumptions:

- (i) the IDR_{DSi} is attained at the first storey;
- (ii) the deformed shape of the ($N-I$) storeys is evaluated as function of their secant stiffness with respect to that expected at the first storey.

Finally, roof displacement $\Delta_{/DSi}$ is divided by the first mode participation factor Γ_1 (ASCE/SEI 41-06, 2007), obtaining SDOF displacement $S_{d/DSi}$.

In the case of DS1 and DS2, the SDOF displacement $S_{d/DSi}$ is evaluated according to equation 11 in which h_{int} is the interstorey height of the building. The IDR of the i^{th} ($i>1$) storey is computed considering the inverted triangular distribution of lateral forces as shown in equation 12, in which H_i and H_j are the heights of the i^{th} and j^{th} storeys above the level of application of the seismic action (foundation or top of a rigid basement). Coefficient γ in equation 11 is the average of the ratio, $\gamma_i=K_1/K_i$, between the stiffness of the first storey (K_1) and that of the i^{th} storey (K_i), all evaluated considering the only infills' contribution and neglecting concrete stiffness contribution at the different storeys.

$$S_{d/DS} = \frac{1}{\Gamma_1} (IDR_{/DS} \cdot h_{int} + \gamma \cdot \sum_{i=2}^n IDR_i \cdot h_{int}) \quad (11)$$

$$IDR_i = IDR_{/DS} \left(1 - \sum_{j=1}^{i-1} \frac{H_j}{\sum_{j=1}^n H_j}\right) \quad (12)$$

For DS1, the stiffness of all the storeys is still elastic, thus leading to $\gamma=1.0$. For DS2 a linear distribution of the stiffness along the height of the building is assumed. Thus, it is evaluated considering the secant stiffness at the first storey (Mainstone, 1971), $K_{sec,Main}$, and elastic at the top storey, K_{el} . $K_{sec,Main}$ was assumed as 25% of the elastic stiffness of the infills at the first storey, leading to the evaluation of γ in equation 13.

$$\gamma = \frac{2K_{sec,Main}}{(K_{el} + K_{sec,Main})} = \frac{2(0.25 \cdot K_{el})}{(K_{el} + 0.25 \cdot K_{el})} = 0.40 \quad (13)$$

For DS3, SDOF displacement $S_{d/DS3}$ is evaluated assuming the same deformed shape for the (N-1) storeys computed for DS2 and displacement increasing is generated by $IDR_{/DS3}$, as shown in equation 14. The latter assumption implies that the unloading stiffness of the (N-1) storeys is infinite.

$$S_{d/DS3} = S_{d/DS2} + \frac{(IDR_{/DS3} - IDR_{/DS2}) \cdot h_{int}}{\Gamma_1} \quad (14)$$

The definition of $S_{d/DSi}$ for DS1, DS2, and DS3 allows the consequent definition of $Sa(T)_{/DSi}$ through the IN2 curve. In fact, $S_{d/DSi}$ is the expression of the characteristic equivalent SDOF displacement that represents the abscissa in the ADRS format in which the IN2 is computed. Thus, the IN2 curve becomes the tool by which $Sa(T)_{/DSi}$ is defined. Given $Sa(T)_{/DSi}$, the switching to $PGA_{/DSi}$ is pursued through a spectral scaling procedure.

6.3 Comparative damage assessment

Two benchmark buildings characterized by 2 and 4 storeys are considered, according to the typical number of storeys characterizing Emilia's RC building stock, and the following assumptions have been made for the computation of DS capacities:

- building area $A_b=300 \text{ m}^2$;
- interstorey height $h_{int}=3.0 \text{ m}$;
- geometric percentage of infilled area respect to the global plan extension $\rho_w=0.017$ in according to data available in Ricci (2010) and Crowley and Pinho (2010);
- κ coefficient, aimed at switching from T_0 to T , equal to 1.40 (Manfredi et al., 2012);
- value of τ_{max} chosen equal to $1.30\tau_{cr}$, according to Fardis (1997) and assuming $\tau_{cr}=0.33 \text{ MPa}$ according to the suggested values of Italian code provisions for clay hollow bricks (Circolare 617, 02/02/2009);
- α and β coefficients considered equal to 0.50 and 0, respectively;
- λ coefficient equal to 0.85 for buildings with more than three storeys and 1.0 otherwise;

- average storey mass normalized by the building area m assumed equal to 0.8 t/m^2 (for residential buildings).

Moreover, for the evaluation of the plastic shear strength of the first storey (V_y), further assumptions have been considered: bays' length $l = 4.50\text{m}$; average concrete compressive strength $f_c = 25 \text{ MPa}$ (Verderame et al. 2001); average steel yield strength $f_y = 370 \text{ MPa}$ (Verderame et al. 2012b), and concrete allowable stress $\sigma_c = 5 \text{ MPa}$.

Capacity curves (CC) and IN2 curves in terms of $S_d(T)$ are shown in Figure 12; while in Table 1 period of the idealized capacity curve, seismic capacity expressed in term of $S_d(T)$ and PGA are reported for each DS.

Table 1. Periods T of the idealized capacity curve, $S_a(T)$ and PGA capacities, evaluated through OPCM 3274 (2003) EC8 (CEN,2004) spectra on D soil class.

Number of storeys N	Period T [sec]	$S_a(T)$ [g]			PGA [g]		
		DS1	DS2	DS3	DS1	DS2	DS3
2	0.148	0.23	0.64	0.73	0.11	0.30	0.35
4	0.296	0.10	0.39	0.56	0.04	0.16	0.22

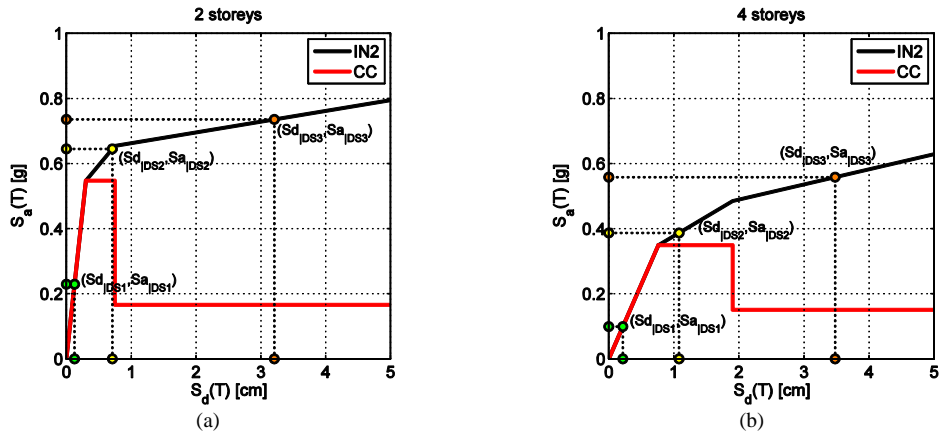


Figure 12. Multi-linearized capacity curves and IN2 curves in terms of $S_d(T)$: (a) 2-storey and (b) 4-storey RC buildings

The results obtained in terms of PGA are compared with the shake map of the 20th of May mainshock provided by INGV (see Figure 4). First of all it should be highlighted that the damage thresholds in terms of PGA shown in Table 1 are computed for the case of soil class D and for the constant shape spectra according to EC8 (type 1). In fact, on the basis of very macroscopic geological scale, most of the area struck by the earthquake is characterized by soil class D.

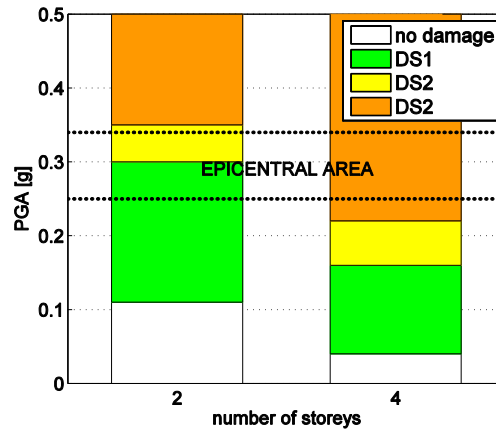


Figure 13. Damage assessment in terms of PGA for 2 and 4 storeys RC buildings, epicentral area is characterized by a PGA range of $[0.25\text{g}-0.34\text{g}]$ according to shake map data (see Figure 4).

The comparison of the numerical PGA thresholds, at the three DSs, and the shake map allows some preliminary remarks. In particular, such remarks are affected by the hypothesis that the considered benchmark buildings are representative of the whole building stock involved by earthquake.

First of all, it is easy to recognize that numerical results predict DS3 damage only in the case of RC buildings localized in the epicentral area and characterized by more than two storeys. According to the numerical data, most of RC buildings should be characterized by DS1 and DS2, with the exception of the epicentral area, see [Figure 13](#). Notwithstanding the fact that RC buildings are not so frequent in the area, numerical results show a fair agreement with data collected during in-field campaigns after the earthquake (e.g., [EPICentre Field Observation Report No. EPI-FO-290512, 2012, Liberatore et al., 2013](#)).

7. CONCLUSIONS

Characteristics of RC building stock that was struck by the Emilia 2012 earthquake and damage observed after the event were analyzed. Data of the Italian National Institute of Statistics are collected and results showed that buildings are characterized by limited number of storeys with more than 60% buildings of two storeys. Age of construction data showed that most of the structures in the area struck by the earthquake were designed up to 80s. Finally, RC buildings represent less than 20% of the total building stock. A focus on the evolution of seismic classification of the Emilia region and results of geological studies led to the conclusion that RC building stock is characterized by low/mid-rise buildings (2-4 storeys) designed for gravity loads only and located on very compressible soil (e.g., soil class D).

Photographic documentation collected in reports and field surveys was employed for a first characterization of damage to RC buildings in the epicentral area. Notwithstanding the significant intensity registered at the closest station to the epicenter during the 20th of May 2012 mainshock (PGA at Mirandola station equal to 0.26g), RC buildings showed slight to medium damage in general, and seldom heavy damage was observed. Only one collapse case was registered in a irregular three-storey building, probably due to brittle failure mechanisms.

First, a code-based seismic assessment is provided through a spectral based approach, leading to code-based bounds for elastic spectral acceleration capacities to be compared with spectra of the records registered at the station closest to the epicenter of the mainshock event. Results of code assessment on RC structures and comparison of spectral acceleration capacities with spectral demand at Mirandola station emphasized a large performance gap of RC buildings that has no counterpart in damage field evidence. This first comparison indicates that masonry infills – which are not taken into account in such a simple code-based procedure – have to be included in the evaluation of seismic performance of existing RC buildings.

Then, a simplified procedure for seismic assessment of infilled existing RC buildings (*FAST*) is described. In *FAST* methodology a simplified capacity curve is constructed based on the assumption of first storey collapse mechanism, and evaluating the contributions of RC and infill elements to the non linear static lateral force-displacement response. Then, the IN2 curve – providing the seismic intensity corresponding to each displacement value – is obtained by means of an R- μ -T relationship accounting for global strength degradation. Displacement capacity at DS1 DS2, and DS3 is evaluated through a mechanical interpretation of damage description provided by EMS-98. The corresponding seismic capacity – that is, the values of seismic intensity (namely, PGA) leading to the attainment of such DSs – are finally obtained through the IN2 curve.

A comparison with the shake map of the mainshock event of the 2012 Emilia sequence explains the limited damage observed to infilled RC structures in the area struck by the earthquake. In fact, only in the epicentral zone the PGA characterizing moderate structural damage and heavy non-structural damage (DS3) was exceeded ([Verderame et al. 2013](#)). Considerations above show how non-structural elements, such as masonry infills, can provide a source of capacity not accounted for in design that can play a crucial role in earthquakes. On the other hand, masonry infills are elements outside the control of codified design procedures. The above observations make the consideration of non-structural elements a fundamental issue for the scientific community and a target for new generation codes.

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